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General Report Session No. 1: Case Histories of Foundations

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Case Histories of Foundations

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RESPONSIBILITY

The General Reporter read every paper and was responsible for writing this report. Two of the Co-Reporters read selected papers and submitted comments to the General Reporter. Time did not permit co-reporters to review this final report which is therefore the sole responsibility of the General Reporter.

INTENT

We assume that these general reports are used to help potential readers locate selected papers of special interest to them. This function requires that papers be subjected to constructive critique, but within the prescribed space limitations, in the context of standards appropriate for a conference of this type, and in recognition of the lack of opportunity for most authors to respond to criticisms.

DISCUSSION OF "USEFUL" CASE HISTORIES

An unfortunate number of the papers in this session are of little value to the profession. Much of the problem seems to be that authors did not give serious thought to how the reader was supposed to benefit from the paper, i.e., to what constitutes a "useful case history".

To the writers, a good case history paper should have most of the following characteristics:

1. The case history should preferably concentrate on one project or on a specific class of problems. A specific project might involve several technical issues whose interaction impacts the case history. A specific problem would involve a single technical problem on various projects, e.g., axial load capacity of untapered piles in clay. A relatively useless exercise is to discuss a series of unrelated projects in a general fashion.
2. Quantitative measurements that show how the full-scale field installation performed. Numerical results are important, not just verbal description.
3. Relevant material properties, dimensions, etc. must be provided so readers can attempt analyses using their own methods. Thus, case histories in geotechnical engineering will generally involve quantitative description of soil properties, layer thicknesses, foundation dimensions, structural loads, etc. Writers should discuss details such as the type of sampler used, whether or not samples were trimmed, storage times, etc.
4. An analysis is not mandatory but in most cases it is nearly so. The field observations are difficult to generalize to other sites and projects unless there is some sort of general predictive scheme. To be most useful, the predictive scheme should be of the type that could be used by any other reasonable practicing engineer.
5. The predicted behavior and field performance should be compared so the reader knows whether the analytical method is likely to be useful for other projects.
6. The authors should either eschew use of local terminology or should define it clearly. Thus, classification of a soil or rock using a national building code is generally unacceptable because world-wide readers are unlikely to have access to the code and thus cannot interpret the classification. Similarly, use of trade names for materials without a clear definition must be avoided.

Unfortunately, only a few papers satisfied all criteria and many satisfied none of them.

THE ORIGIN OF PAPERS

Fifty seven papers were accepted for this session. The distribution of papers by country of origin of the authors (below) indicates surprisingly limited contributions from some countries where the level of technical sophistication should have led to well documented case histories.

Country	Papers
India	17
USA	14
China	8
Japan, Poland, Egypt	2
Canada, Croatia, France, Great Britain, Hong Kong, Iran, Mexico, Nigeria, Saudi Arabia, Taiwan	1

TOPICS

The range of topics was considerable as indicated in Table 1.

Table 1 - Distribution of Papers by Topic

General Topic	Subclassification	No. of Papers
Driven Piles	Axial Load Tests in Soil	4
	Lateral Loading	3
	Settlement	1
	Driving Formulas	1
	Wave Equation Analyses	1
	General	3
	Auger Cast	1
	Pore Pressure Generation	1
Drilled Piers	Axial Load Tests in Soils	2
	Axial Load Tests in Rock	2
	Lateral Load Tests in Soil	1
	General	2
Shallow Foundations	General	1
	Expansive Clays	4
	Settlement	3
	Bearing Capacity Tests	1
	Plate Bearing Tests	1
	Raft Foundation	1
	Tilt Remediation	3
	Dissolvable Soils	1
	Consolidation	1
Screw Anchors	Load Tests	1
In Situ Ground Treatment		3
Embankment Stability		1
Dynamic Soil-Structure Interaction		1
Drainage		1
Structural Design		3
Geology		1
General		2
Tunneling		1
Scour		1
Monuments		2
Probability		1
Excavation Problems		1
Rock Mechanics		1

DRIVEN PILE FOUNDATIONS

Axial Load Tests

Compression tests. **Miller and Lutenegger (1.39)** reported on the results of load tests on three 60-mm diameter open ended pipe piles driven from 3 to 11 m into a varved clay. They used a one-year set-up time and measured the strength of the clay using a field vane. The authors used the API RP-2A α-method to predict pile capacities. The ratio of predicted pile capacities to measured capacities ranged from 1.4 for shortest pile to 2.0 for the longest. The authors conclude that use of post-peak vane strengths leads to improved predictions. The authors ignore a number of factors including: (1) the recommendation by Dennis and Olson (cited in the paper) that vane strengths be multiplied by 0.7 prior to using the API method, (2) the fact that set up times for piles in the data base were usually about a week whereas their set up times were a year, and (3) the fact that they prebored a desiccated crust.

Tension tests. **Davie et al. (1.24)** indicate that many agencies in the USA follow the BOCA code in defining failure in a tension test on a pile as occurring when the tip movement is 2.5 mm (0.1 inch). They reported results of uplift tests on 25 piles at 15 sites and conclude that the BOCA requirement is too conservative. They suggest a less conservative alternative. It is unfortunate that they did not provide more quantitative information on soil properties and depth to water table so persons interested in static analyses could also have used the data.

Axial loading of a bent pipe. **Dunlop et al. (1.8)** were involved in a project in which 1300 TPT piles (406-mm diameter shafts and enlarged tip) were driven. The piles were driven through 13 m (42 feet) of normally consolidated flyash and silt and were thus likely to be surrounded by softened soil, perhaps even a gap. One pile was driven to a depth of about 15 m (50 feet) with a tip lateral displacement of about 61 cm (24 inches). The pile was instrumented with an inclinometer and loaded axially to twice the design load of 1.35 MN (150 tons). Lateral deflections were computed using the Winkler approximations using several simplified analytical approaches as well as using STRUDL. A variety of approximations were used for the coefficients of subgrade modulus but the final results seemed not sensitive to the values chosen. The analyses correlated well with the measurements and indicated that the pile failed at about twice the design load.

Long Term Pile Settlement

Rico et al. (1.64L) discuss the problem of regionally subsiding soils on the design of pile foundations in Mexico City. The goal is to provide a foundation such that the piles do not emerge from the ground due to surface settlements. Numerous typing errors and omissions make it difficult to track the analyses. It is unclear how the authors calculated side shearing strengths, and how group effects were taken into account. Some statements, such as "even within a few days, the soil adherence becomes fixed", seem contrary to worldwide experience.

Surface Settlement from Pile Driving and use of Auger Cast Piles

Leznicki et al. (1.14) present field data indicating substantial surface settlements when open ended pipe piles were driven through about 27 m (90 feet) of sand to shist bedrock in New York City. A 52-story structure was to be pile supported at a site, with a historical building, that must be protected from damage, on shallow foundations on the adjacent property. After driving a number of open ended pipe piles and noticing significant surface settlements, they switched to 400 mm (16-inch) diameter by 27 m (90 feet) long auger cast piles and installed such piles as close as one meter (4 feet) from the historical building. Special procedures were required to minimize lost ground for the auger cast piles. Installation of the auger cast piles led to settlements of the historical building of up to 38 mm (1.5 inches). Settlement data were presented for the tower for 80 weeks.

Lateral Loading

Yudhbir and Basudhar (1.61) report an effort to predict lateral deflections of a pile group in silt due to lifting of a heavy tank. The lateral pile capacity was calculated in accord with equations recommended by Broms and group effects were modeled using interaction factors recommended by Poulos. Although the foundation was instrumented during the lift, the authors were denied access to the data and thus could not confirm the accuracy of their computations except based on the observation that the lift was successful.

Raju et al. (1.69) reported failure of some 1-m diameter cast-in-place concrete piles for a "berth" at an offshore site off the west coast of India. The piles penetrated 15 m of water, 4 m of soft clay, and 1 m into rock. The authors use finite element analyses and field measurements to show that the natural frequency of the free-standing casings were similar to the frequency of wave action. They recommend that the casings be braced prior to concrete set up.

Luong (1.73L) plucked some 63 kV steel lattice work overhead line towers and reduced the resulting vibration data in an attempt to draw conclusions about possible deterioration of the foundations. No supporting field evidence of such deterioration or the success of the method is included.

Pore Pressures Generated by Pile Driving

Chandra and Hossain (1.66) present a useful paper on pore pressure changes induced by driving I shaped precast concrete piles in soft Bangkok clay. Their tests at the Asian Institute of Technology campus site involved pore pressure measurements at radii between 0.4 and 4 m and at depths from 3 to 9 m. Data are reported during pile installation and over a three month dissipation period, and are compared with predictions made using a critical state, one dimensional, cavity expansion analysis. The Cambridge finite element code, CAMFE, was used, with input parameters being taken from an associated laboratory study. Parametric studies were performed to help interpret the data and improve the degree of fit. The authors were able to obtain reasonable agreement for the magnitudes of pore pressures during driving, but found it difficult to match subsequent decay processes accurately. The authors do not discuss problems with cavity expansion models as applied to driven piles, nor the problem of using CAMFE when the soil properties vary radially due to varying amount of shearing during pile driving. They may find that the strain path method, proposed by Baligh, will help them. A literature review of recent field experiments may also be helpful.

Descriptive Papers

Daniels et al. (1.41) summarize extensive pile driving experiences for a bridge across the Mississippi River north of St. Louis, Missouri. They used H piles with reinforced tips to penetrate dense sands, gravels, and occasionally cobbles and boulders. They also used composite pipe and H piles. They used a Hydroblok hammer and used the Pile Driving Analyzer and CAPWAP analyses to predict pile capacities. The paper is grossly overlength and so full of information that would not usually be relevent to a paper that the major points are lost.

Dynamic Pile Analyses

CAPWAP and PDA analyses have been mentioned by authors of papers cited previously, e.g., 1.41, and will not be repeated.

Hussain and Sheahan (7.36L) report the results of eleven load tests and 32 CAPWAP analyses for precast concrete piles at five sites along the route of a bridge in Fort Myers, Florida. They report that the actual hammer delivered energy only averaged 48% of the rated energy and that injection of cooling water into the pile cap during driving reduced apparent hammer efficiency by 5-10%. They found that the uplift side capacity was 76% of the side capacity in compression (the later based on CAPWAP). They found that the pile capacities increased linearly on a log time plot for 42 days (no DATA points on the plots). The authors do not provide specific quantitative data on piles and soil conditions at each site to allow the reader to interpret the case histories independently.

Senapathy et al. (1.42) used GRLWEAP retap analyses to show that pile capacities at one site increased by 1.5 to 3 times in 24 hours. The subsoil consisted of interstratified sand, silt, and clay. They claimed that static analyses based on NAVFAC and Meyerhof agreed reasonably well with "measured values". The authors provide no useful information to allow independent analysis of their data. Their "measured" capacities seem to involve significant extrapolations in many cases, e.g., piles P1, S1, S4, and S5.

Techman and Gwizdala (1.43) used various dynamic formulas, none of which were presented, to predict capacities of steel pipe piles in sand and sandy gravel for a crane foundation in Poland. Both the pile driving formulas and their static analysis overpredicted pile capacities. They provide no useful information on the piles, or soil conditions at the site, nor even on their analytical methods.

DRILLED PIERS (DRILLED SHAFTS, BORED PILES, CAISSONS)

Axial Capacity

Alsamman and Long (1.30) compared predicted and measured axial load capacities of nineteen drilled piers, using three methods that are based on quasi-static cone tests. The methods came from Nottingham, Laboratoire des Ponts et.Chaussees (LPC), and Poulos and Davis. Only tests in which the soil profile was all sand or all clay were considered. Failure was defined to occur at a settlement of 5% of the butt diameter plus the elastic shortening of the pier. The LPC method was the most successful with a range in the ratio of calculated to measured capacities (Q_c/Q_m) of 0.8 to 1.4 for piers in clay and 0.3 to 1.9 in sand.

Datye and Patil (1.54) discuss an effort to relate chisel behavior with the axial load capacity of piers that were constructed using chisels and bailers in Bombay. The method was unsubstantiated with field data and was apparently not very successful since the authors report that failures had occurred.

Lateral Loading

Benvie and Kirby (1.44) outline the procedures they use in the New York City area for the design of single drilled pier foundations under cellular telephone monopoles. For lateral loading (main problem) they use the p-y approach and program LPILE1 from Reese and Wang, with the p-y curve empirically related to standard penetration test results. They indicate that pole behavior has been satisfactory for two years but they provide no field measurements of actual pole behavior.

Lutenegger and Miller (1.38) report data from load tests on 0.51-m diameter rigid drilled piers at their campus site at the University of Massachusetts, where the subsoil is a soft varved clay with a desiccated crust. They predicted lateral deflections using the p-y method and program LPILE1, with soil properties coming from prebored monocell pressuremeter and dilatometer tests. Predicted and measured deflections were in reasonable agreement. It would have been interesting for them to discuss the extent of drainage in the varved clay during their tests. With only four #6 reinforcing bars, one wonders if the piers cracked during loading. No information is provided on toe displacements but such short piers should have been influenced by shear and moments at the toes. It is unfortunate that the authors did not report predictions based on the more commonly used p-y curves estimated from strength correlations (Reese, Matlock, O'Neill, and others).

Piers in Rock

Trow et al. (1.10) discuss the performance of 750 to 900 mm diameter drilled piers, socketed into rock, for a 48-story condominium on the shores of Lake Ontario in Canada. They cast one drilled pier into rock with a space between the tip and the rock so as to generate only side shear but were unable to cause the pier to fail and finally decided on a design side shear of 533 kPa in weathered rock and 1 MPa in unweathered rock. They also report a test for end bearing where a reduced diameter tip was able to sustain 72 MPa successfully after undergoing failure at 128 MPa. During construction they tried to air lift debris prior to tremi-pouring of concrete but later borings showed rubble in the bottom of the piers. Soil borings near one pier showed that concrete had flowed as far as 4.7 m from the pier hole. Low amplitude integrity tests showed the presence of either poor concrete or intruded soil in some piers, which were subsequently repaired. The foundation design assumed that the piers would transfer all of their load in side friction in the rock sockets. Field strain gage monitoring showed: (1) that the total loads were higher than expected, (ii) that skin friction in the overburden took 20% of the applied load, and (iii) the foundations performed acceptably. One problem with the paper was that terms "settlement" and "strain" were sometimes used inappropriately. Further, load test details are illegible in the figures and not covered in the text. No information is provided on rock quality, strength, or stiffness.

Panozzo et al. (1.47) present the results of two load tests on drilled pier socketed into limestone bedrock. They placed strain gages on reinforcing bars to allow computation of load transfer and used styrofoam under one pier to eliminate end bearing. The authors present minimal descriptions of rock properties. They also presented theoretical solutions for load transfer in rock based on the theory of elasticity and also some empirical correlations. Rock modulii were backed out of load test data. This paper is grossly overlength.

Kesavanathan and Kozera (7.30) present data for pier foundations for the 26-story IBM building in Baltimore. The piers passed through dense sand and weathered rock into an amphibolite. Two piers were instrumented but the instrumentation in one was destroyed. At the end of construction, the load-deflection curve for the instrumented pier indicated that it was in the elastic range so no data exist on failure stresses. Calculated local deflections were based on an assumed rock modulus, the origin of which is not stated. However, the side shear in a dense layer (particle size ranging from silt to gravel, N=44 to 100/3) was 1.3 ksf which exceeds the upper limit of 1.0 ksf set in the U.S. Navy design manual (NAVFAC DM7).

General

Rao (1.16) discusses a new foundation type consisting of "solid bored piles" with diameters of a meter or two and lengths generally less than 100 m, installed either by rotary drilling or percussion, using drilling mud. Problems develop in 1% to 2% of the foundations from construction problems. The discussion in the paper indicates that the author considers this a new foundation type. However, it is unclear how these foundations differ from common drilled piers.

SCREW ANCHORS

We will use the term "screw anchor" for a shaft containing one or more single auger flights, that is screwed into the ground.

Seider (1.37) discusses load tests on three such anchors that were designed for use underpinning foundations. The anchors were screwed into stiff clay at an angle, from the edge of a grade beam. Because the anchors were tilted, the vertical loading caused a moment in the anchor rods. Moments were measured in the shafts as a function of depth. Lateral loads between the shafts and the soil were also computed using program LPILE and were generally smaller than measured values. No curves of load versus deflection were provided so the reader does not know whether the anchors were close to failure or not. No design method was advocated nor supported by the tests.

BEHAVIOR OF SHALLOW FOUNDATIONS

Osinubi (1.13) reports that measured settlements of buildings in the Ukraine constructed on hydraulic fills overlying "soft alluvial deposits" were slightly less than values computed using equations from Florin and Tsytovich. The equations are not provided and no data are provided to allow the reader to perform independent analyses so the conclusions will be mainly of interest to engineers already familiar with recommendations by Florin and Tsytovich.

Sinha (1.27) reports a case in India where a large number of single story houses were to be renovated by adding a second story. Plate bearing tests were used, together with some formulas from an Indian code, to conclude that some structures could be expanded using the same foundations and some could not. The author does not indicate whether the subsoils beneath the plates and footings were assumed to be drained or undrained and presents no quantitative data for independent analyses.

Dong, Qian, and Huang (1.46) present field measurements of settlements of a library extension in Shanghai. Unfortunately, little information is provided on required soil properties and no effort is made to compare computed and measured movements.

Gryczmanski and Sekowshi (1.58) discuss a five-story building constructed with shallow foundations, with part of the building underlain by peat with water contents up to 400%. The structure underwent settlements exceeding 200 mm and experienced distortions over 1:70. In the absence of data on soil properties, loadings, and geometry, the case cannot be analyzed. They are considering underpinning the structure.

Aoki et al. (1.76) analyzed data from a raft foundation at a depth of 17-18 m and underlain by gravel and silt. Pressures applied to the subsoil by the raft varied from 230 kPa to 343 kPa. Peak heave during excavation was about 42 mm and settlements were 25 to 33 mm. The authors measured low strain elastic modulii of the subsoils geophysically, corrected for differing strain levels, treated the raft as being spring supported, and used an ill-defined iterative technique to calculate settlements. Measured settlements were about 1.2 to 1.3 times the computed settlements.

Viladkar and Saran (1.22) reanalyzed an existing foundation of a petroleum vacuum distillation tower in India to take additional load. The subsoil was stratified silt and clay. The authors present a single plot of settlement versus root time from a laboratory consolidation test and in spite of the fact that it shows that Terzaghi's theory does not apply, they apparently used Terzaghi's theory to calculate that the degree of consolidation of the subsoil under the existing foundation, after 17 years, was 78%. They used an Indian code limit on tilt of 1:400 and a Bjerrum correlation of distortion to total settlement to set a limit on total settlement of 120 mm. They then determined the additional load to limit the ultimate settlement to their calculated value. The authors present no data on the tilt of the existing structure in spite of the fact that their computations are based on limiting the tilt. They apparently used Terzaghi's theory for a case in which it could not apply (layered system, three dimensional drainage, soil does not consolidate in accord with the theory). They do not consider bearing capacity. They present no field data of actual performance.

Jardine et al. (1.40) constructed five rigid concrete footings, with side lengths of 2.2 to 2.4 m, at a depth of 0.8 m, at a site in Scotland. The subsoil is lightly cemented, brittle, clayey silt to clay, with a thin shelly layer. They reported strength measurements involving a variety of sampling and testing methods. Mean undrained shearing strengths for a depth range of 206 m ranged from 9 to 28 kPa. Footings were loaded with kentledge during a one to four day period. One pad was loaded to failure in 80 hours and another was loaded to 2/3's of the failure load and observations mad over a continuing period of time. Field measurement included lateral deflection, pore water pressures, and settlement at several depths. Under steady loading, the settlement-log(time) curve became linear after about 80 hours and remained so out to 28 months although excess pore water pressures had dissipated after about 14 months. The authors conclude that there was partial drainage even during a two to three day loading period but that failure was progressive. Numerous more detailed observations were made, and are included in a number of referenced companion papers. The work warrants serious study.

MISCELLANEOUS TOPICS

Monuments

Sharma (1.23) reports a case in which a temple in India, originally constructed in the Twelfth century, was partially dismantled with the intent to move it, but then reassembled at the same location.

Rao et al. (1.70) report on efforts to determine the geotechnical conditions beneath the famous Taj Mahal in India, apparently to assess the possibility of damage resulting from raising the level of the adjacent river slightly. Based on consolidation testing, the authors estimate that the structure has settled 141 cm and that consolidation is now 99.4% complete, after 350 years of loading. It seems unfortunate that no effort was reported to measure differential movements within the structures to see if they were consistent with the predicted settlement

Drainage

Chang and Wu (1.9) discuss the problem of hydrostatic uplift on basements of major structures and indicate that usual solutions in Taipei involve using tie downs or adding weight to the foundation. They chose to surround the walls and slab of a major building with a geocomposit drainage layer and drain the water into tanks in the basement and then into the sewers. They do not discuss the problem of settlement of surrounding buildings due to the decreasing water table.

Correction of Tilt

Structures (buildings, tanks, towers, etc.) periodically rotate after construction because of such diverse causes as: (1) designs that ignored the presence of soils of non-uniform compressibility, and (2) non-uniform loading. The critical question then becomes one of deciding what to do. Several authors addressed this question, sometimes in an innovative fashion.

Shi-Tae et al. (1.34) report that numerous buildings in Wuhan, China, have tilted and then been corrected. They report that people generally complain when the tilt exceeds about 0.5% (1:200). They reported on cases where the tilt ranged from 1% (1:100) to 2.5% (1:40). In some cases where the tilt is believed to result from plastic movements in the subsoil, they drive piling adjacent to the structure to minimize these movements. In other cases, they try to apply extra load to the side that has settled less. In a more innovative mode, they drilled holes adjacent to the high side of a structure and caused the soil, at depth, to flow into the holes, thus removing soil from beneath the structure and causing it to rotate. In one case they caused one side of a 7-story building to settle 230 mm by drilling 38 holes to a depth of 9-10 m.

In Madras, India, **Sridharan and Murthy (1.49)** used a variety of techniques, including driving angled piling under footings, adding load to one side, and lowering the water table locally. More innovatively, in cases where the subsoil was silty (erodable), they drove perforated pipes and pumped water out of the pipes. Water flowing in through the perforations eroded the silt locally and caused settlement.

Amiesoleymani (1.65) encountered tilt problems with two grain elevators in Iran. The elevators settled up to 450 mm and dragged down one edge of adjacent lightly loaded structures that were used for loading and emptying the elevators. Rotation of the adjacent structure then caused the adjacent structure to strike the elevator structure and risk serious structural damage. Ameriesoleymani used buried chains to cut out horizontal slices, about 25 mm thick each, of the subsoil under parts of the structures to cause additional settlement and elimination of tilt. Unfortunately, the details of the cutting procedure were not covered but the technique seemed innovative and was apparently successful.

Local Geology

Jain (1.18) discusses local geology in the area of New York City and reviews several consulting projects to show the effect of varied geology on foundation design.

Dissolvable Soils

Fatani and Khan (1.20) discuss the case of a town constructed largely on top of a salt dome. Water leaking from fresh water and septic tanks causes the salt to be dissolved and to undermine the buildings. Some buildings have collapsed. The authors suggest several obvious solutions but present no case histories involving actual application of the suggested methods.

Scour

Jain, Saxena, and Bhargava (1.26) report that two bridges with shallow foundations failed due to scour. They recommend that soil exploration be performed prior to construction and that the superstructure should be appropriate to the foundation conditions.

Excavation Problems

Soric et al. (1972) report a variation on the usual theme in which an excavation is made next to an adjacent structure and causes foundation movements. In their case, a building was demolished, thus unloading its foundation and causing problems for the adjacent building. They used a diaphragm wall for support of the adjacent structure. They were not able to provide analytical data for stability and did not discuss stability when the trench for the diaphragm wall was open.

General

Saxena et al. (1.57) provide a discussion of several foundation problems in Hyderabad, India. Little quantitative information is provided. The conclusions are that soils investigations should be performed and that investigators of failures may be biased.

EXPANSIVE CLAYS

For inexplicable reasons, engineers in many parts of the world continue to have unexpected problems with expansive clays. The problems usually result from placing structures on shallow foundations on top of desiccated clays and then introducing water via irrigation, leaking sewer or fresh water lines, or from impeded surface drainage. Alternatively, the structure is placed on drilled piers with grade beams and there is inadequate clearance between the grade beams and the expansive clay (**Reyad, #1.7**)

Qian (1.51) dealt with a slightly expansive clay in China by using sand fills with thicknesses of 0.3 to 1.0 m, placing an impervious apron out 0.8 to 2.0 m from the buildings, and keeping water pipes exposed. Such techniques are in wide use and seem helpful when properly designed.

Zunjing and Mei (1.52) encountered problems with desiccation of expansive clays in southern China in "red clay". They minimized the shrinkage problem by using a 2-3-m wide concrete-over-sand apron around houses, sometimes placing sand under house foundations, and sometimes placing house foundations as deep as 1.5 m.

El-Sohby and Elleboudy (1.56) reported desiccation problems with a site that had previously been irrigated. They recommended that irrigation near the structures be stopped (would seem to exacerbate the problem), that the subsoil be grouted, and that surface water be deflected away from the structures.

STRUCTURAL PROBLEMS

Several papers (**Rao (1.12) Viswanath (1.35), Miglani (1.71)**) dealt with structural problems involved with design of floating bridge caissons, pile caps, and a stadium roof, and did not involve geotechnical problems directly.

EMBANKMENT STABILITY

Mohan (1.4) reported the failure of a preloaded "stacking ground" adjacent to a wharf. The stacking ground was made of a fill over clay with sand drains used to accelerate consolidation. Stability analyses "without any surcharge" gave factors of safety of 0.9 to 1.5 based on vane strengths. There were several small failures during construction. In the absence of quantitative data on soil properties, loads, etc. it is difficult to draw useful conclusions from this "case history".

IN SITU TREATMENT

Wu, Chen, and Feng (1.2) improved the properties of a 15-m deep soft clay in China, for support of a surface pressure of 300 kPa from an oil tank, using "Mono-axis deep mixing". The columns were 0.55 m in diameter and 1.05 m on center and extended to the full depth of 15 m. They indicate that they raised the bearing capacity and reduced settlement but present no data on performance of the full scale tank to substantiate the success of their treatment.

Chummar (1.21) reported that four spherical petroleum storage tanks were constructed on a site in India where there was soft clay (c_u=0.03 to 0.3 kg/cm2 from field vane) to a depth of 11 m. Stone columns were used for support but the tanks settled excessively when empty and one failed by rotary motion during water testing. The stone columns had a peak measured capacity of 40 tonnes for one pile and 135 tonnes for three. Columns were 90 cm in diameter and 1.2 m on center. Failure occurred for a mean footing pressure of 12 tonnes/sq.m. (17 tonnes/column). The author seems to believe that the strength of the clay was lower than originally measured, thus causing the failure. He does not explain how the measured capacity of a column could be so much higher than the apparent load when the tank failed.

Hsu (1.75L) apparently used some undefined explosive technique to compact a collapsible loess and then pile supported a large oil tank. The author reports that the tank is working well.

TUNNELING

Lee et al (1.19) present a summary case study of the ground response which developed during soft ground tunneling below the water table in Shanghai, China. The tunnel was 4.2 m in diameter at a depth of 5.6 m. Excavation was accomplished with an Earth Pressure Balance Shield with instrumentation to determine in situ earth and pore water pressures at the excavated face. Site instrumentation permitted measurements of pore pressure changes and horizontal and vertical displacements to be made, and provided insight into variations in the context of real construction activity records. The authors suggest use of a face support pressure 30% above the at rest values. The companion analytical studies consider settlements due to the stress changes due to tunneling but seem not to consider the effect of the tunnel acting as a drain. The authors discuss the importance of including, in the analyses, the effects of a disturbed zone with dimensions deduced from the results of pore pressure measurements. Immediate and consolidation settlement contributions are separable on the basis of pore pressure measurements and changes with time. Settlement magnitudes and the dimensions of the settlement trough are very close in measurements and from modeling to the settlement patterns proposed by Peck (1969). The authors do not provide information on the type of piezometer nor the date of piezometer installation.

PROBABILITY

There has long been an interest in trying to represent scatter in soil properties using appropriate probabilistic techniques. Such efforts have been limited by two particular facts, viz.: (1) most of the sources of error are systematic and result from such effects as sampling disturbance, and (2) most of the spatial variation in soil properties is actually the result of systematic variations in site geology which may not be understood by the designers based on a limited number of borings.

In any case, **Ochiai et al. (1.33)** examined the variations in side shear between 1.2 m diameter drilled piers and surrounding soil in terms of the scatter in standard penetration resistance's (N). A pair of load tests on one pier gave ratios of side shear/N that differed significantly from the Japanese standard for clays. For one clay layer, f_s was only 60% of that expected whilst in another f_s was 2.5 times higher. However, the pier was not carried even close to apparent failure. The main conclusions for the reviewers was that simple design rules based on SPT 'N' values are unreliable. Nevertheless, the authors performed statistical kriging analyses to consider the consequences of spatial variations in N values and came to the unsurprising conclusion that "the uncertainties of the estimated N-values were affected by distance of soil investigations." Readers in the USA should realize that Japanese N values differ significantly from the U.S. standard values.

DYNAMIC SOIL-STRUCTURE INTERACTION

The paper by Jiang and Zhao (1.5) was mainly concerned with analysis of dynamic soil-structure interaction and probably belonged in a different session of this conference.

ROCK MECHANICS

When stability problems developed in some lead/zinc mines in Hindustan at depths around 300 m, **Rajmeny and Sinha (1.62L)** noticed that boreholes drilled at that depth were becoming elliptical due to breakout and thus that there was an anisotropic state of horizontal stress. They used hydrofracting and overcoring to show that the minor horizontal stress was 70% of the major horizontal stress. They concluded that fracturing began at a strength/stress ratio of 1.2 and collapse for ratios of 1.0 or less. They attempted to develop a technique for estimating the needed support based on the breakout in the boreholes. The review copy has no figures, making some of the discussion difficult to follow.

SUMMARY AND CONCLUSIONS

The discussion of almost all of the papers was critical because few satisfied the majority of items that should be included in a useful case history. On a more positive note, the General Reporter will take it upon himself to recommend certain papers.

In the area of deep foundations, **Miller and Lutenegger (1.39)** present well documented axial load test data for open ended pipe piles in clay.

Dunlop et al. (1.8) demonstrate a useful way of analyzing axial load capacity of a bent pile.

Leznicki et al. (1.14) present a good case history involving settlements in loose sand due to pile driving and use of an alternative foundation type.

Chandra and Hossain (1.66) present interesting data on pore water pressures generated around piles and their time rate of dissipation.

Hussain and Sheahan (7.36L) provide some interesting information relating to the relative magnitudes of side shear in tension and compression.

Alsamman and Long (1.30) provide useful summary information on comparison of measured and predicted capacity of drilled piers and recommend a specific approach.

Lutenneger and Miller (1.38) provide a well documented case history on short drilled piers under lateral loading.

Trow et al. (1.10), **Panozzo et al. (1.47)**, and **Kesavanathan and Kozera (7.30)**, all provide fragmentary information on rock socketed drilled piers. The data will be particularly useful to persons working in the same geologic formations.

Jardine et al. (1.40) presented a well documented case history of load tests on rigid concrete footings in clay. Numerous references provide detailed information that could not be included in the paper.

Amiesoleymani (1.65) presented an interesting and innovative approach to the righting of tilted structures. **Shi-Tae et al. (1.34)** and **Sridharan and Murthy (1.49)** suggest other alternatives.

Lee et al (1.19) present important data on pore pressures in soft clays due to tunneling and to the resulting settlements.

The lack of useful case histories on such classic topics as settlement of building foundations in sands and clays was unexpected.